Illinois Department of Transportation

Memorandum

To:	ALL BRIDGE DESIGNERS	03.7
From:	Ralph E. Anderson	Ralph E. Anderson
Subject:	LRFD – Shear Stud Design	
Date:	October 1, 2003	

This memorandum is the fifth in a series detailing the Department's policies and procedures for implementation of the AASHTO LRFD Bridge Design Specification by October 1, 2007.

This bureau has developed a design example to illustrate the Department's procedure for design of shear stud connectors welded to steel girders. This design example is based on the LRFD Specifications and modifications as developed by the Department.

The design of shear studs is based on both fatigue and strength limit states. Both of these checks should be made in determining the number and spacing of shear studs. Connectors shall only be placed in positive moment areas for new structures. Field splices of stringers should be located such that shear studs are not placed on the top flange splice plate. If this can not be avoided, in no case shall a stud be placed within 3 inches of the transverse centerline of the field splice.

In determining the dead load contraflexure location, the designer should only use beam and slab dead load (DC1) and superimposed dead load (DC2) and should not include the future wearing surface dead load (DW).

Additional connectors required by the Specifications shall be located within a distance envelope equal to1/3 the effective flange width, either side of the point of dead load contraflexure or centered over it. These connectors are in addition to those required by fatigue and strength limit states.

Article 6.10.7.4.2 requires the number of cycles (N) to be used in calculating the fatigue resistance of shear studs. For a "No Truck Route", N shall be set to one (1). Using the value of zero (0) will yield an error in the equation.

The current AASHTO Standard Specification's requirement extending the negative reinforcement bars in the top slab a distance of 40 bar diameters beyond the anchorage connectors, shall be continued on all LRFD designs. This is a deviation from the LRFD Specifications that states reinforcement shall extend a distance equal to the development length past the shear connectors.

The following example is for a symmetrical three span plate girder bridge. It illustrates the design and placement of shear stud connectors in all spans.

Additionally, revised Interior Girder Moment and Reaction Tables for use on LRFD projects have been included with this memorandum.

SYK/ZBU/03.7/bb25360

LRFD Stud Shear Connector Design AASHTO 1998 2nd Edition w/ 99-03 Interims

For Bridge Geometry, Design Stresses and Traffic Data, see Figures 1 and 2.
For Plate Girder Dimensions, Dead Loads and Section Properties, see Figure 3.

From previous calculations, the Contraflexure Points (DC1+DC2) were found to be:
Span 1 - 55'-9 1/2" (from Abutment)
Span 2 - 21'-3 1/2" (from each Pier)
Span 3 - 23'-2 1/2" (from Pier 2)

Article 6.10.7.4 - Shear Connectors

Article 6.10.7.4.1a - Types - For stud connectors, $\frac{H}{d} \geq 4.0$

$$H_{min} = 4.0\text{''},\ d = 0.75\text{''}$$

$$\frac{H}{d} = \frac{4.0\text{''}}{0.75\text{''}} = 5.33 \geq 4.0 \ \therefore \text{OK}$$

Article 6.10.7.4.1b - Studs shall be designed for Fatigue Limit State, then check for Strength Limit State

$$p \leq \frac{nZ_r I}{V_{sr} Q}$$

$$p_{min} = 6d = 6 \times 0.75\text{''} \Rightarrow \ p_{min} = 4.5\text{''}$$

$$p_{max} = 24\text{''}$$

Article 6.10.7.4.1c - Transverse Spacing and Edge Distance

$b_f = 12\text{''}$

Min Transverse Spacing $= 4d = 4 \times 0.75\text{''} \Rightarrow \ S_{trans,min} = 3.0\text{''}$

Min Edge Distance $= 1.0\text{''}$

Min distance to centerline of stud $= 1.0\text{''} + 0.5(0.75\text{''}) = 1.375\text{''}$

The number of studs per row, n, that can be used with the above restrictions is 3.

Article 6.10.7.4.1d - Cover and Penetration

Clear depth of concrete over the studs shall not be less than 2.0" and the studs shall penetrate the deck at least 2.0"

Article 6.10.7.4.2 - Fatigue Resistance of Shear Connectors in Composite Sections

$$Z_r = \alpha d^2 \geq \frac{5.5d^2}{2}$$

$\alpha = 34.5 - 4.28 \log N$ N - Number of Cycles Specified in Article 6.6.1.2.5

Use N=1 for "No Truck Routes"

$N = 365 \times 75n(ADTT)_{SL}$

n - cycles per truck passage

$n = 1.0$ From Table 6.6.1.2.5-2 Span Lengths > 40ft and "Elsewhere"

$(ADTT)_{SL} = p \times ADTT$ $p = 0.85$(2 Design Lanes) From Table 3.6.1.4.2-1

$$(ADTT)_{SL} = 0.85 \times \frac{206}{2} = 87.55 \qquad \text{use } (ADTT)_{SL} = 88 \text{ Trucks For Single Lane}$$

$$N = 365 \times 75 \times 1.0 \times 88 \;\Rightarrow N = 2,409,000 \text{ Cycles for 75yr Design Life}$$

$$\alpha = 34.5 - 4.28\log(2,409,000) \Rightarrow \alpha = 7.19\,\text{ksi}$$

$$Z_r = 7.19\text{ksi} \times \left(0.75''\right)^2 = 4.04\,\text{kips} \geq \frac{5.5 \times \left(0.75''\right)^2}{2} = 1.55\,\text{kips} \Rightarrow Z_r = 4.04\,\text{kips}$$

Span1, 3-Determine Required Pitch for Fatigue Truck Loading

$$p \leq \frac{nZ_rI}{V_{sr}Q} \quad \text{with } n = 3,\ Z_r = 4.04\,\text{kips } I = 27,370\,\text{in}^4 \text{ and } Q = 615\,\text{in}^3$$

$$p \leq \frac{3 \times 4.04 \times 27,370}{V_{sr} \times 615} = \frac{539.4}{V_{sr}}$$

Tenth Pt		
0.0	$V_{sr} = 26.2\,\text{kips} \Rightarrow p = 20.6''$	The maximum pitch of 24" is violated everywhere but
0.1	$V_{sr} = 22.5\,\text{kips} \Rightarrow p = 24.0''$	0.0 to 0.1 Span 1. The minimum pitch of 4.5" is ok.
0.2	$V_{sr} = 19.8\,\text{kips} \Rightarrow p = 27.2''$	
0.3	$V_{sr} = 18.1\,\text{kips} \Rightarrow p = 29.8''$	
0.4	$V_{sr} = 16.8\,\text{kips} \Rightarrow p = 32.1''$	
0.5	$V_{sr} = 17.2\,\text{kips} \Rightarrow p = 31.4''$	
0.6	$V_{sr} = 18.2\,\text{kips} \Rightarrow p = 29.6''$	
0.7	$V_{sr} = 19.6\,\text{kips} \Rightarrow p = 27.5''$	
0.706	$V_{sr} = 19.6\,\text{kips} \Rightarrow p = 27.5''$	

Article 6.10.7.4.4 - Strength Limit State

$$n = \frac{V_h}{Q_r}$$

n is the number of connectors needed between point of maximum positive and each adjacent point zero moment. (0.0L to 0.4L and 0.4L to 0.706L)

$$Q_r = \phi_{sc}Q_n \qquad \phi_{sc} = 0.85 \qquad \text{Article 6.5.4.2}$$

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \qquad \text{Article 6.10.7.4.4c}$$

$$E_c = 33,000w_c^{1.5}\sqrt{f'_c} = 33,000 \times 0.15^{1.5}\sqrt{3.5} \Rightarrow E_c = 3587\,\text{ksi} \qquad \text{Article 5.4.2.4}$$

$$Q_n = 0.5 \times 0.4418 \times \sqrt{3.5 \times 3587} = 24.75\,\text{kips} \leq 0.4418 \times 60 = 26.51\text{kips use } 24.75\,\text{kips}$$

$$Q_r = 0.85 \times 24.75\,\text{kips} \Rightarrow Q_r = 21.04\,\text{kips}$$

Article 6.10.7.4.4b - Nominal Horizontal Shear Force, V_h

The total horizontal shear force between the point of maximum positive moment and each adjacent point of zero moment. V_h shall be taken as the lesser of the two following equations.

$V_h = 0.85f_c^{'}bt_s = 0.85 \times 3.5 \times 79 \times 7.5 = 1,763\,\text{kips} \Leftarrow$ Governs

or

$V_h = F_{yw}Dt_w + F_{yt}b_t t_t + F_{yc}b_c t_c = 50 \times 36.375 = 1,819\,\text{kips}$

$V_h = 1,763\,\text{kips}$

Required Spacing to Satisfy Strength Limit State, n - # of studs

$$n = \frac{V_h}{Q_r} = \frac{1,763}{21.04} \Rightarrow n = 84\,\text{studs} \quad \#\text{rows} = 28$$

Spacing: 0.0L to 0.4L (31.6'): $p_1 = \frac{L_1}{\#\text{Spaces}} = \frac{31.6 \times 12}{27} = 14.04"$ use $p_1 = 14"$

Spacing 0.4L to 0.706L(24.174'): $p_2 = \frac{L_2}{\#\text{Spaces}} = \frac{24.174 \times 12}{27} = 10.74"$ use $p_2 = 10.5"$

The controlling pitch is for Strength.

Note: Eq. 6.10.7.4.4b-3 is not used because beams are non-composite over the interior supports (Negative Moment Regions)

Article 6.10.7.4.3 - Special Requirements for Points of Permanent Load Contraflexure

$$n_{ac} = \frac{A_r f_{sr}}{Z_r} \qquad Z_r = 4.04\,\text{kips}$$

n_{ac} - # of additional connectors

A_r - total area of reinforcement within the effective flange width

Top - #5 at 12" cts and #6 at 12" cts $\Rightarrow A_{r,top} = 4.94\,\text{in}^2$

Bottom - 6-#5 Bars Between Beams $\Rightarrow A_{r,bot} = 1.86\,\text{in}^2$

$A_r = 6.80\,\text{in}^2$

Article 5.5.3.1 f_{sr} - stress range in the longitudinal reinforcement

$$f_{sr} = \frac{M_r}{S_{top}}$$

Determine Section Modulus of Steel Beam plus Longitudinal Reinforcement Assume 0" fillet

$$y_b = \frac{A_{st} y_{b,bm} + A_{r,top} y_{b,top} + A_{r,bot} y_{b,bot}}{A_{st} + A_r} = \frac{36.375 \times 21.75 + 1.86 \times 45.44 + 4.94 \times 47.5}{36.375 + 1.86 + 4.94}$$

$$y_b = 25.71"$$
$$y_t = 21.79"$$

$$I_c = \sum (I + Ad^2)$$

Steel Section $I_c = I_x + A_s (y_b - y_{b,bm})^2 = 10,923 + 36.375(25.71 - 21.75)^2 = 11,493\,\text{in}^4$

Bottom Bars $I_c = A_{r,bot} (d + d_b - y_b)^2 = 1.86(43.5 + 1.94 - 25.71)^2 = 724\,\text{in}^4$

Top Bars $I_c = A_{r,top} (d + t_s - d_t - y_b)^2 = 4.94(43.5 + 7.5 - 3.5 - 25.71)^2 = 2,346\,\text{in}^4$

$$I_c = 14,563\,\text{in}^4$$

$$S_{top} = \frac{I_c}{y_t} = \frac{14,563}{21.79} \Rightarrow \qquad S_{top} = 668\,\text{in}^3$$

M_r - Fatigue Truck Moment Range at Point of Contraflexure

$M_r = 158.7 - (-93.8) \Rightarrow \quad M_r = 252.5\,\text{k-ft}$

$$f_{sr} = \frac{M_r}{S_{top}} = \frac{252.5 \times 12}{668} \Rightarrow \quad f_{sr} = 4.54\,\text{ksi}$$

$$n_{ac} = \frac{A_r f_{sr}}{Z_r} = \frac{6.80 \times 4.54}{4.04} = 7.64 \qquad \text{use } n_{ac} = 8$$

The additional connectors shall be placed within one-third of the effective flange width on either side of the point of contraflexure. Field splices shall be placed so that shear studs will not be placed on top flange splice plate. See Figure 3 for stud layout.

Span2 – Determine Required Pitch for Fatigue Truck Loading

$$p \le \frac{nZ_r I}{V_{sr} Q} \quad \text{with } n = 3,\ Z_r = 4.04\,\text{kips}\ I = 29,586\,\text{in}^4 \text{ and } Q = 652\,\text{in}^3$$

$$p \le \frac{3 \times 4.04 \times 29,586}{V_{sr} \times 652} = \frac{549.97}{V_{sr}}$$

Tenth Pt	0.201	$V_{sr} = 22.6\,\text{kips} \Rightarrow p = 24.3''$	The maximum pitch of 24" is violated everywhere.
	0.3	$V_{sr} = 21.0\,\text{kips} \Rightarrow p = 26.2''$	The minimum pitch of 4.5" is ok.
	0.4	$V_{sr} = 20.3\,\text{kips} \Rightarrow p = 27.1''$	
	0.5	$V_{sr} = 20.2\,\text{kips} \Rightarrow p = 27.2''$	
	0.6	$V_{sr} = 20.3\,\text{kips} \Rightarrow p = 27.1''$	
	0.7	$V_{sr} = 21.0\,\text{kips} \Rightarrow p = 26.2''$	
	0.799	$V_{sr} = 22.6\,\text{kips} \Rightarrow p = 24.3''$	

Article 6.10.7.4.4 - Strength Limit State

$$n = \frac{V_h}{Q_r}$$

n is the number of connectors needed between point of maximum positive and each adjacent point zero moment. (0.201L to 0.5L and 0.5L to 0.799L)

$Q_r = \phi_{sc} Q_n \qquad \phi_{sc} = 0.85 \qquad$ Article 6.5.4.2

$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \le A_{sc} F_u \qquad$ Article 6.10.7.4.4c

$$Q_n = 0.5 \times 0.4418 \times \sqrt{3.5 \times 3587} = 24.75\,\text{kips} \le 0.4418 \times 60 = 26.51\text{kips} \;\; \text{use } 24.75\,\text{k}$$

$$Q_r = 0.85 \times 24.75\,\text{kips} \Rightarrow Q_r = 21.04\,\text{kips}$$

Article 6.10.7.4.4b - Nominal Horizontal Shear Force, V_h

The total horizontal shear force between the point of maximum positive moment and each adjacent point of zero moment. V_h shall be taken as the lesser of the two following equations.

$$V_h = 0.85 f'_c b t_s = 0.85 \times 3.5 \times 79 \times 7.5 = 1{,}763\,\text{kips} \Leftarrow \text{ Governs}$$

or

$$V_h = F_{yw} D t_w + F_{yt} b_t t_t + F_{yc} b_c t_c = 50 \times 37.875 = 1{,}894\,\text{kips}$$

$$V_h = 1{,}763\,\text{kips}$$

Required Spacing to Satisfy Strength Limit State

$$n = \frac{V_h}{Q_r} = \frac{1{,}763}{21.04} \Rightarrow n = 84\,\text{studs} \qquad \#\text{rows} = 28$$

Spacing: 0.201L to 0.5L (31.694')

$$p_1 = \frac{L_1}{\#\text{Spaces}} = \frac{31.694 \times 12}{27} = 14.09'' \;\; \text{use } p_1 = 14''$$

Spacing 0.5L to 0.799L(31.694') $$p_2 = \frac{L_2}{\#\text{Spaces}} = \frac{31.694 \times 12}{27} = 14.09" \text{ use } p_2 = 14"$$

The controlling pitch is for Strength.

Article 6.10.7.4.3 - Special Requirements for Points of Permanent Load Contraflexure

$$n_{ac} = \frac{A_r f_{sr}}{Z_r} \qquad Z_r = 4.04\,\text{kips}, A_r = 6.80\,\text{in}^2$$

Article 5.5.3.1 f_{sr} - stress range in the longitudinal reinforcement

$$f_{sr} = \frac{M_r}{S_{top}}$$

Determine Section Modulus of Steel Beam plus Longitudinal Reinforcement Assume 0 fillet

$$y_b = \frac{A_{st} y_{b,bm} + A_{r,top} y_{b,top} + A_{r,bot} y_{b,bot}}{A_{st} + A_r} = \frac{37.875 \times 21.01 + 1.86 \times 45.56 + 4.94 \times 47.62}{37.875 + 1.86 + 4.94}$$

$$y_b = 24.97"$$
$$y_t = 22.65"$$

$$I_c = \sum (I + Ad^2)$$
Steel Section $I_c = I_x + A_s (y_b - y_{b,bm})^2 = 11,608 + 37.875(24.97 - 21.01)^2 = 12,202\,\text{in}^4$
Bottom Bars $I_c = A_{r,bot} (d + d_b - y_b)^2 = 1.86(43.625 + 1.94 - 24.97)^2 = 789\,\text{in}^4$
Top Bars $I_c = A_{r,top} (d + t_s - d_t - y_b)^2 = 4.94(43.625 + 7.5 - 3.5 - 24.97)^2 = 2,535\,\text{in}^4$

$$I_c = 15,526\,\text{in}^4$$

$$S_{top} = \frac{I_c}{y_t} = \frac{15,526}{22.65} \Rightarrow \qquad S_{top} = 685\,\text{in}^3$$

M_r - Fatigue Truck Moment Range at Point of Contraflexure
$M_r = 142.1 - (-78.6) \Rightarrow \quad M_r = 220.7\,\text{k-ft}$

$$f_{sr} = \frac{M_r}{S_{top}} = \frac{220.7 \times 12}{685} \Rightarrow \quad f_{sr} = 3.87\,\text{ksi}$$

$$n_{ac} = \frac{A_r f_{sr}}{Z_r} = \frac{6.80 \times 3.87}{4.04} = 6.51 \qquad \text{use } n_{ac} = 7$$

The additional connectors shall be placed within one-third of the effective flange width on either side of the point of contraflexure. Field splices shall be placed so that shear studs will not be placed on top flange splice plate. See Figure 3 for stud layout.

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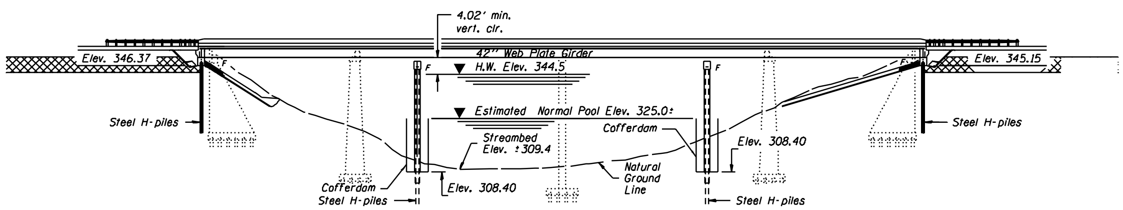
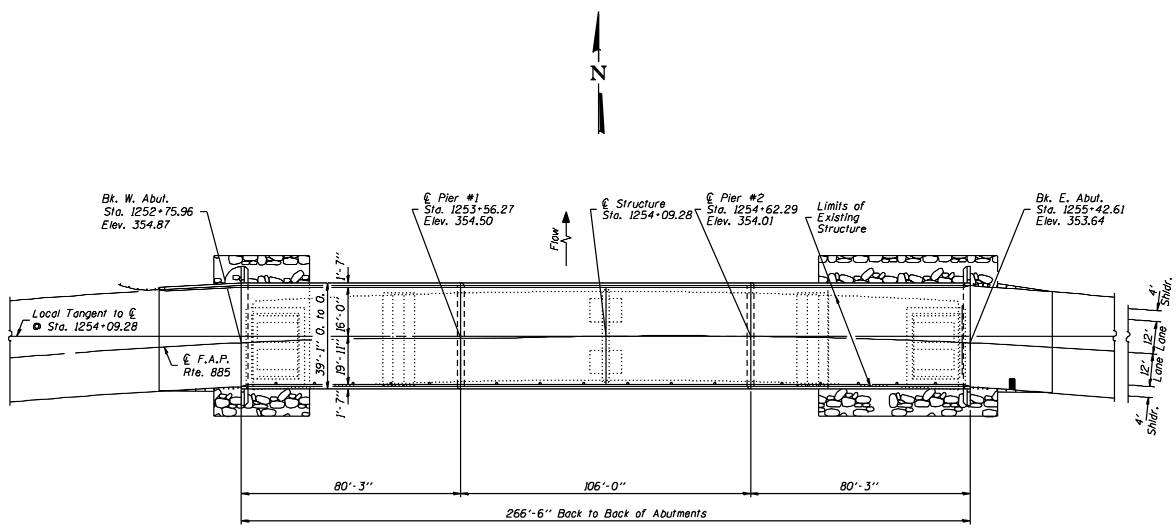
SHEET NO.
SHEETS



ELEVATION



PLAN

LOADING HL-93

Allow 50#/sq. ft. for future wearing surface.

DESIGN SPECIFICATIONS

1998 AASHTO LRFD with 1999 thru 2003 Interims

DESIGN STRESSES

FIELD UNITS

f'_c = 3,500 psi
f_y = 60,000 psi (reinforcement)
f_y = 50,000 psi (structural steel, M270, Gr. 50)

TRAFFIC DATA

ADT = 3425 (Future)
% Trucks = 6%
ADTT = 206
2 Design Lanes

DESIGNED	
CHECKED	
DRAWN	
CHECKED	

2002
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Figure 1

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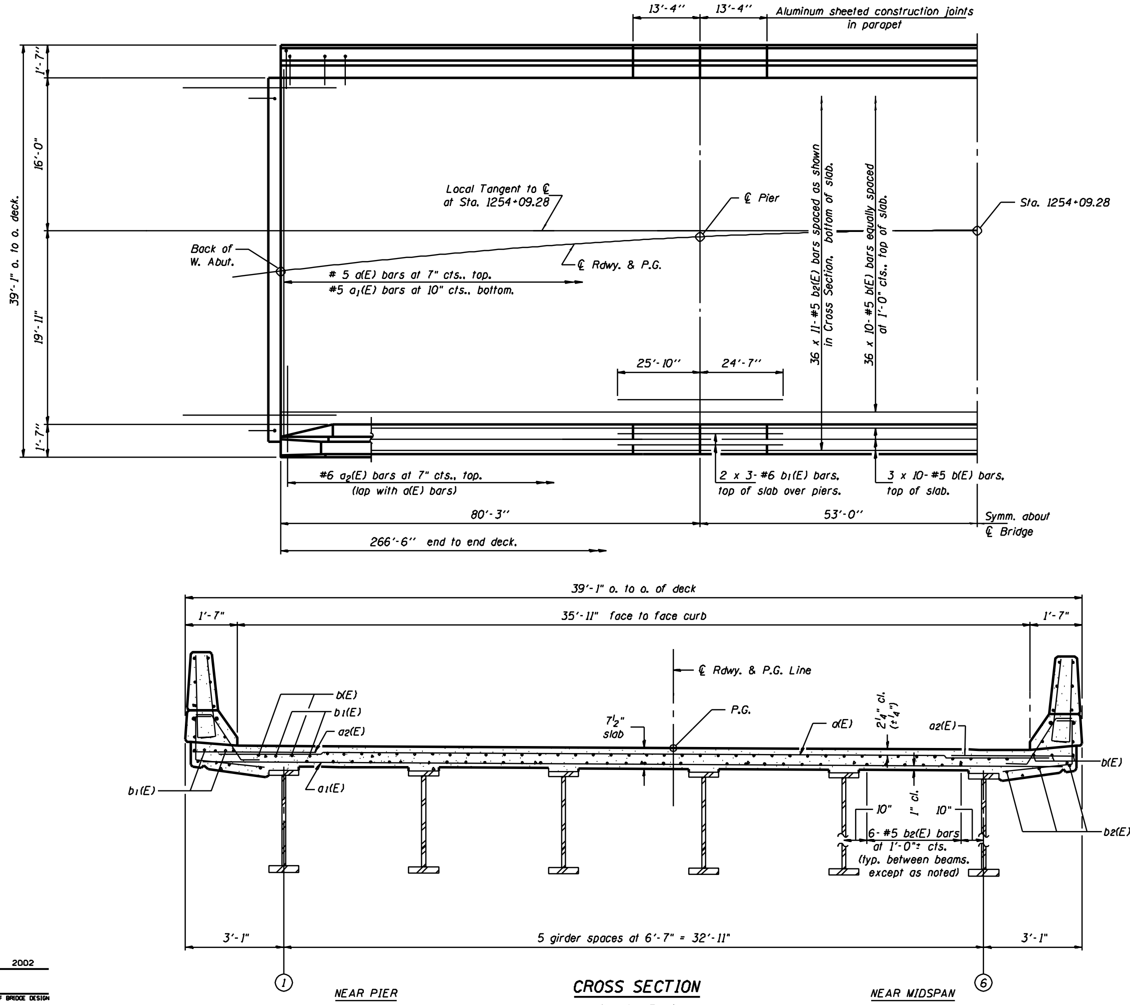

CROSS SECTION
(Looking East)

DESIGNED	
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DRAWN	
CHECKED	

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SHEETS

Figure 2

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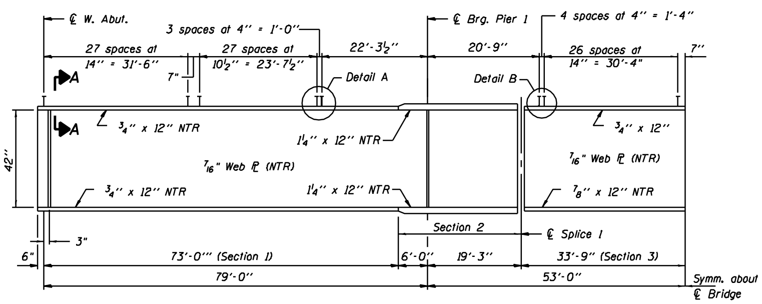
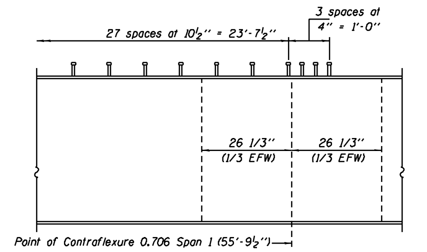


GIRDER ELEVATION



DETAIL A

Additional studs must be within 1/3 of EFW around point of contraflexure. On either side or centered about it.

SECTION 1

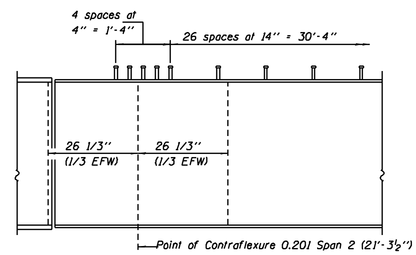
DC1 = 0.778k/ft
Ix = 10923 in⁴
Ic(n) = 27370 in⁴
Q = 615 in³

SECTION 2

DC1 = 0.827k/ft
Ix = 16731 in⁴

SECTION 3

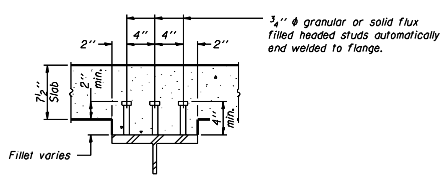
DC1 = 0.784k/ft
Ix = 11608 k/ft
Ic(n) = 29586 in⁴
Q = 652 in³



DETAIL B

Additional studs must be within 1/3 of EFW around point of contraflexure. On either side or centered about it.

DC1 calculated with 1'' fillet assumed
DC2 (Parapet) = 0.150 k/ft
DW (F.W.S.) = 0.333 k/ft



SECTION A-A

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Figure 3

Standard Interior Girder Moment and Reaction Table for LRFD Designed Superstructure and LFD/ASD Designed Substructure

Interior Girder Moment Table - 2 Span Symmetrical			
		0.4Sp1 or 0.6Sp2	Pier 1
Is	in^4	15587	37778
Ic(n)	in^4	41658	-
Ic(3n)	in^4	30111	-
Ss	in^3	816	1608
Sc(n)	in^3	1127	-
Sc(3n)	in^3	1034	-
Z	in^3	-	-
DC1	k/ft	0.922	1.094
M DC1	k-ft	606	1802
DC2	k/ft	0.137	0.137
M DC2	k-ft	107	215
DW	k/ft	0.333	0.333
M DW	k-ft	260	522
M LL+Imp	k-ft	1512	1494
Ma (Strength I)	k-ft	3927	5919
Mr	k-ft	5922	-
fs DC1	ksi	8.9	13.4
fs DC2	ksi	1.2	1.6
fs DW	ksi	3.0	3.9
fs 1.3(LL+I)	ksi	20.9	14.5
fs (Ser II)	ksi	34.1	33.4
fs (Total)(Strength I)	ksi	-	44.2
Vsr	k	21.6	-

Interior Girder Reaction Table - HL93 Loading		
	Abutment	Pier
R DC1	34.0	141.8
R DC2+DW	18.6	64.5
R LL	68.5	128.9
R Imp	22.6	42.5
R Total	143.7	377.7

Interior Girder Reaction Table - HS20 Loading		
	Abutment	Pier
R DL	52.6	206.3
R LL	44.2	78.4
R Imp	9.5	16.8
R Total	96.8	284.7

(To be used with LFD designed substructures)

Is and Ss are the moment of inertia and section modulus of the steel section used in computing fs due to non-composite loads.

Ic(n) and Sc(n) are the moment of inertia and section modulus of the composite section used in computing fs due to short-term composite loads.

Ic(3n) and Sc(3n) are the moment of inertia and section modulus of the composite section used in computing fs due to long-term composite loads.

Z is the plastic section modulus used to determine the fully plastic moments in the non-composite areas.

DC1 is the dead load acting on the non-composite section.

DC2 is the dead load acting on the long-term composite section.

DW is the dead load acting on the long-term composite section due to wearing surface.

Ma (Strength I)=1.25 M(DC1+DC2)+1.5 M DW +1.75 M(LL+Imp)

Mr is the full plastic moment capacity computed in accordance with 6.10.3.1.3 and 6.10.4.2.

fs (Service II) is the sum of the stresses due to DC1+DC2+DW+1.3(LL+Imp)

fs (Total) (Strength I) (Non-Compact Section) is the sum of the stresses due to 1.25(DC1+DC2)+1.5DW+1.75(LL+Imp)

Vsr is the maximum shear range in the span (0.75 LL+Imp)